I - 19 END RESTRAINT OF EXTENDED END-PLATE CONNECTIONS ON IN-PLANE FRAME ANALYSIS

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1. Introduction

Conventional methods of steel frame analysis employ two highly idealized connection models: the fully rigid connection model and the pinned connection model. Since the actual behavior of connections always falls in between these extreme cases, modern specifications allow the incorporation of semi-rigid connection in frame analysis. Extended end-plate connection, a typical configuration of which is shown in Fig. 1, is very often used as a rigid connection to sustain high moments. The aim of this research is to study the level of rigidity of real extended end-plate connections by executing in-plane frame analyses. A number of experimental moment-rotation curves (112) for extended end-plate connection as described in the data base of Kishi et al. (1994) are used in frame analyses. Frame responses obtained from frame analyses for real extended end-plate connections are compared with those of fully rigid connections. The comparison is conducted for different frame spacing.

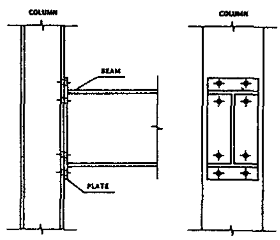

Fig. 1 A typical connection

2. Frame Analyses

Second-order non-linear elastic frame analyses as proposed by Goto and Chen (1978) have been conducted to examine the degree of rigidity of real extended end-plate connections. A 2-bay 2-story frame is chosen for the numerical analyses as illustrated in Fig. 2. Element nos. and node nos. are shown in the boxes and circles respectively. Beam-column sections and length are kept constant for all frame analyses , while frame spacing was varied from 150 inch to 300 inch with an increment of 50 inch. Frame responses are calculated both for the real extended end-plate connections and for fully rigid connections. Responded values for real connections are normalized by the responded values for rigid connections. Normalized drift d^* under service load and normalized end moment m^* under factored load are then plotted against initial connection stiffness R_{ki}. These figures are drawn for different frame spacing. Applied load intensities are listed in Table 1 and the load combinations (service load: D+L+W, factored load: 1.2D+0.5L+1.3W) are chosen according to AISC-LRFD (1986) specification.

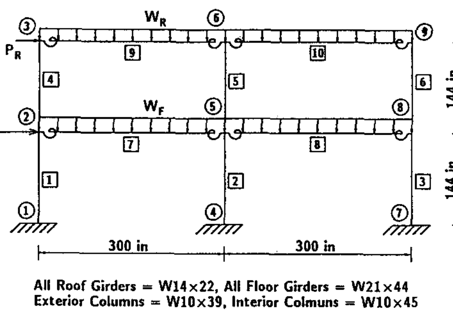

Fig. 2 2-bay 2-story frame

3. Results of Frame Analyses

The connections having low R_{ki} is disregarded from the rigid connection consideration. In this study, minimum R_{ki} for rigid connection is taken as 10^6 kip-inch/radian.

Table 1 Applied Load Intensity

Load	Dead (D)	Live (L)	Wind (W)
Floor	68 psf	48 psf	20 psf
Roof	20 psf	20 psf	

3.1. Drift Under Service Load

Figure 3 shows an example of d^* vs. $\log_{10}R_{ki}$ under service load obtained by numerical analysis for node 3 of the frame with 200 inch frame spacing. It is clear from the figure that the connections having an initial connection stiffness more than 10^6 kip-inch/radian, are mostly (77.8%) clustered in the $0.90<d^*<1.10$ region. The same is true for other frame spacings. Results of drift analyses for four different spacings are tabulated in Table 2. It is obvious from the table that, percentage of rigid connections has decreased as the frame spacing is increased. The implication here is clear, with decreasing frame spacing, the same connection can be estimated as more rigid and its d^* can be approximated to unity.

Table 2 Numerical Results of Drift under Service Load

Frame spacing	No. of data of which $R_{ki}>10^6$kip-in/rad.	No. of data of which $0.90<d^*<1.10$	% of rigid data in $0.90<d^*<1.10$
150 in	45	39	86.7
200 in	45	35	77.8
250 in	45	34	75.6
300 in	45	34	75.6

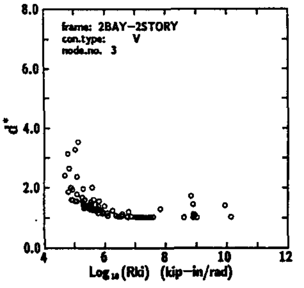

Fig. 3 Drift under service load

3.2. Moment under Factored Load

An illustrative example of m^* vs. $\log_{10}R_{ki}$ for node 3 of element 9 are represented in Fig. 4. In this figure, it is obvious that the connections having $R_{ki}>10^6$ kip-inch/radian are mostly clustered in the vicinity of $m^*=1.0$ line. Table 3 represents the summerized results of frame analyses for four different spacings. It seems that frame spacing has less influence on beam end moment compared to drift. There is virtually no change of percentages of rigid connections for end moments of beams 9 and 10. Only in beam 7, a distinct reduction of percentage of rigid connection with frame space increment is noticed.

Table 3 Numerical Results of Moment under Factored Load

Elem. no.	Node no.	% of rigid data in $0.90<m^*<1.10$			
		150 in	200 in	250 in	300 in
7	2	95.6	93.3	84.4	84.4
	5	84.4	77.8	75.6	75.6
8	5	77.8	77.8	71.1	71.1
	8	97.8	100	97.8	97.8
9	3	100	97.8	97.8	97.8
	6	100	100	100	100
10	6	100	100	100	100
	9	100	100	100	100

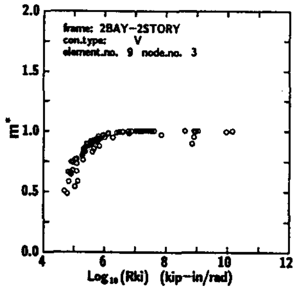

Fig. 4 Moment under factored load

4. Conclusion

All real extended end-plate connections are not as rigid as fully rigid connection. A good number of them have an initial connection stiffness less than 10^6 kip-in/radian. These connections can be regarded as semi-rigid connections. Other connections whose initial connection stiffness are greater than 10^6 kip-in/radian can be classified as rigid connections depending upon their ultimate moment capacity. Frame analyses in this study reveal that most of the connections having high initial connection stiffness resembles with the characteristics of fully rigid connection. Moreover, frame spacing has an influence on the degree of rigidity of the same connection. The closer the frame spacing is the more rigid the connection is.

5. References

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